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Research Paper

Effects of cyclic shear stress and average shear stress on the cyclic loading failure of marine silty sand

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ABSTRACT

This paper presents the cyclic direct simple shear (CDSS) tests to explain the long term cyclic behavior of marine soil deposits. Cyclic behavior of marine soil deposits depend on the number of loading cycles, vertical effective stress, cyclic shear strain amplitude, relative density, and cyclic and average shear stresses. In addition, the study investigates the effect of cyclic and average shear stresses on the cyclic behavior of marine soil deposits subjected to long term cyclic loading. Design graphs are plotted from the CDSS test results. These results are then modeled and can be used to design offshore structures such as offshore wind turbine foundations, and also used to assess the cyclic shear strength of soil beneath the foundation.

1. Introduction

1.1 Background and objective

The cyclic behavior of marine soil deposits has been an important topic over the past few decades. Goulois et al. (1985) studies the effect of average sustained shear stresses on the reduction of the modulus of clay due to repeated cyclic shear loading otherwise known as cyclic degradation. Andersen et.al (1988) provided general knowledge about soil behavior under cyclic loading and constituted a database which can be used to determine soil parameters for early feasibility studies of gravity structures on clay. Andersen (2009) explained how soil behaves under cyclic loading, and presented diagrams with cyclic shear strength of clay, silt, and sand. In the design of offshore wind turbines, various methods have been used to include loading in the design procedure. One such method is called the design graphs or design contours, which accounts for the effects of the Cyclic and Average Stress ratios on the mode of failure of soil by degradation. These graphs are based upon cyclic triaxial or cyclic simple shear tests (Nielsen et al., 2012). The aim of this study is to perform cyclic simple shear tests on marine soil deposits to obtain the cyclic ratios and cyclic strains at failure in order to produce design graphs for the construction of offshore wind turbine foundation at West Sea, South Korea.

1.2 Cyclic direct simple shear test

The cyclic direct simple shear test is a laboratory procedure which applies a vertical confining stress onto a soil sample and the average shear stress and cyclic

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shear stress can be controlled by a moving tray by the user. The shear stress is induced throughout the height of the sample using a wire-reinforced membrane as shown in Fig. 1. The test can either be strain-controlled or force-controlled depending on the requirements of the test.

In this paper, the suitability of the cyclic direct simple shear (CDSS) over the other methods lies in the fact that compared with the cyclic triaxial test, the CDSS captures symmetrical loading with almost no permanent shear strain (Fig. 2). The experimental device used in this device is an NGI Type Direct Simple Shear Device as described in the paper of Bjerrum and Landva (1966).

In the triaxial test, the shear strain is asymmetric with a permanent shear strain of the same magnitude as the cyclic shear strain. This is related to the strength anisotropy under triaxial loading, with an extension strength that is smaller than the compression strength. (Andersen, 2009).

Hence, for the purposes of this paper, the CDSS is most suitable since it can display clearly the strain type (permanent or cyclic) to the appropriate and corresponding character of the shear stress loading (symmetric and asymmetric). The graphical representation of said concepts can be seen in Fig. 3.

Moreover, the CDSS replicates the undrained condition as seen in marine silty sand foundations. Rather than measuring pore pressures, which would require complete saturation of the sample, the pore pressure response is inferred from the change in vertical stress which is monitored throughout the test Baxter et al. (2010). In this way, the change in vertical stress ($\Delta\sigma v$) to keep the sample height constant are assumed to be equal to the excess pore water pressure (Δu) that would develop if the test were truly undrained with actual pore pressure measurements. (Dyvik et al., 1987).

1.3 Design graphs

Design graphs are visual representations of the average and cyclic shear stress components for cyclic behavior of soils as shown in Fig. 4. The design contour also shows the proportion of permanent to cyclic shear strains accumulated by the soil sample throughout the indicated number of cycles to failure. Hence, the design graph captures the combined effects of average and cyclic shear stresses to the mode of failure of the soil as well as the prevailing strain type at the instance of failure. The applied cyclic and average shear stress ratios are usually normalized by the initial vertical confining stress and are respectively known as the cyclic stress ratio (CSR) and the average shear stress ratio (ASR).

Table 1. Summary of the CDSS tests with corresponding CSR and ASR values.

Test ID	Cyclic Stress Ratio ($\tau_{cyc} / \sigma`_{vc}$)	Average Stress Ratio ($\tau_a / \sigma`_{vc}$)
CDSS_01	0.2	0.00
CDSS_02	0.2	0.20
CDSS_03	0.2	0.30
CDSS_04	0.2	0.50
CDSS_05	0.3	0.00
CDSS_06	0.3	0.20
CDSS_07	0.3	0.30
CDSS_08	0.3	0.50
CDSS_09	0.4	0.00
CDSS_10	0.4	0.20
CDSS_11	0.4	0.30
CDSS_12	0.4	0.50
CDSS_13	0.5	0.00
CDSS_14	0.5	0.30
CDSS_15	0.5	0.50
CDSS_16	0.6	0.20
CDSS_17	0.75	0.00
CDSS_18	0.75	0.20

Table 2. Marine silty sand properties.

Properties of Soil Tested		
Min. Void Ratio, e_{min}		0.74
Max. Void Ratio, e_{max}		1.18
Coefficient of Uniformity, C_u		1.8
D_{10}	(mm)	0.08
D_{30} (mm)		0.09
D_{60}	(mm)	0.14
USCS		Silty Sand (SM)
Specific Gravity, G_S		2.62

Detailed evaluation of the importance of the average and cyclic shear stress components for cyclic behavior is given by (Andersen, 1998) on plastic Drammen Clay. Randoph and Gourvene (2011) also produced such contour diagrams and presented results of the asymmetric cyclic simple shear test performed on normally consolidated Drammen Clay. As for the DSS applicability in non-cohesive soils Kammerer et al. (2004) extensively used CDSS in testing the Nevada Sand for the construction of the PEER in UC Berkeley.

1.4 Failure criteria

In selecting a suitable failure criterion we have to see the different proposals in literature. In the study of Ishihara (1985) he found that the initial occurrence of liquefaction is in the strain range of 2.5 % to 3.5 % and

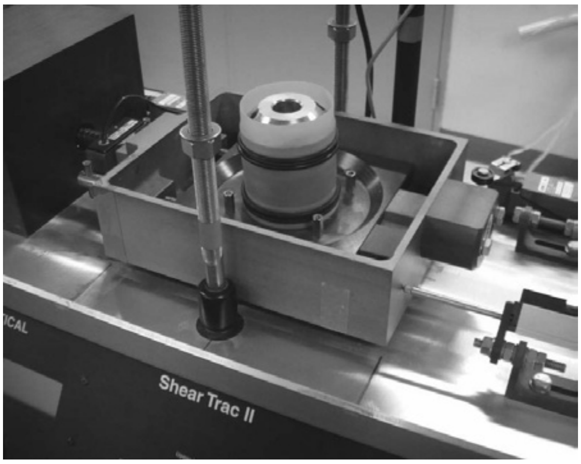

Fig. 1. Cyclic direct simple shear (CDSS) apparatus with soil sample in wire-reinforced membrane.

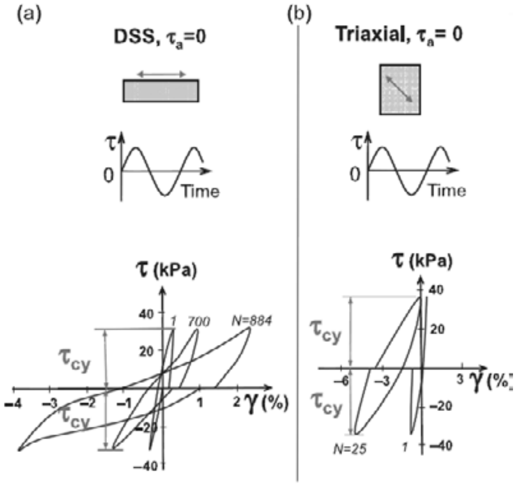

Fig. 2. Comparison of cyclic strain response of cyclic direct simple shear test to cyclic triaxial test to symmetric shear stress loading (Andersen, 2009).

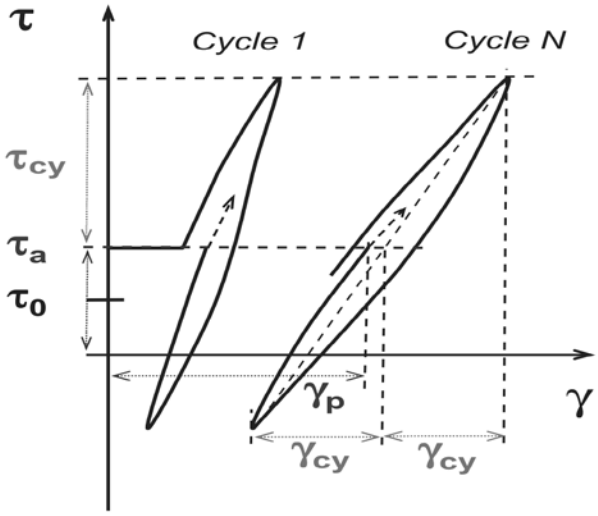

Fig. 3. Visualization of the two kinds of shear stresses (cylic and average) and the two kinds of strains (cylic and permanent) Andersen (2009).

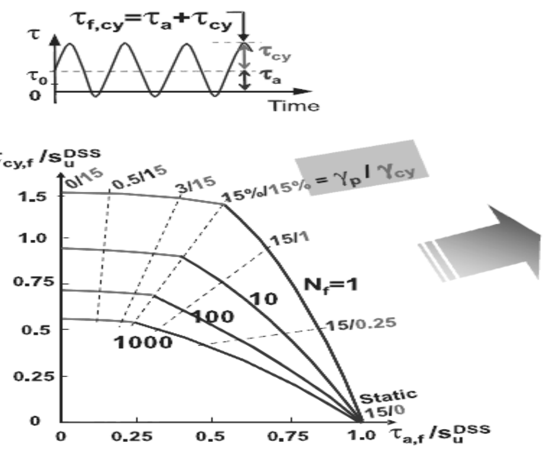

Fig. 4. Sample design graph indicating cyclic and average stress ratios as well as the cyclic and permanent strains. Andersen (2009).

therefore recommended a 3 % single amplitude strain. On the other hand, the study of Seed defined the failure criteria at 7.5 % double amplitude strain De Alba et.al (1976). Lastly in the study of Nielsen et al. (2012) on the Frederikshavn sand, the authors defined the failure criteria as a double amplitude strain of 15 %. It must be noted that the experiments done by Neilsen were intended for the design of offshore structures. Hence, it is with this mindset that a 15 % double amplitude strain is used in this study.

2. Testing program

The 18 CDSS Tests are performed at a frequency of 0.1 Hz. All the tests are performed at relative density of 85 %. The failure criteria of 15 % cyclic double amplitude shear strain or permanent shear strain is selected for all the tests. A relative density of 85 % was the initial relative density before consolidation step. To produce in-situ (Ko) stress conditions, a vertical consolidation stress must be applied to the sample prior to shearing. Applied vertical stresses simulate the loads from overburden material located over the soil sample. For marine silty sand, a normal consolidation stress of 200 kPa is applied in two steps for all specimens.

In this study marine silty sand is obtained from the West coast of South Korea. Specific gravity of material tested is Gs = 2.62. Marine silty sand has minimum voids ratio of 0.74 and maximum voids ratio of 1.18. Details of properties of soil tested are given in Table 2.

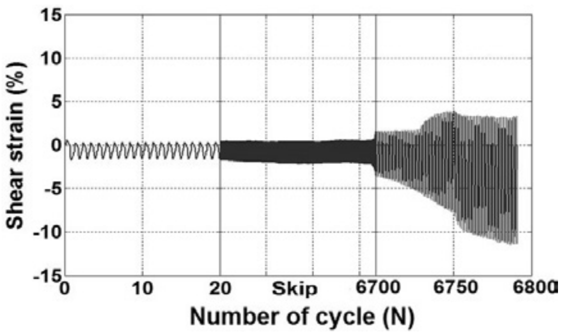

Fig. 5a. Shear Strain versus number of loading cycles with CSR = 0.3 at 15 % double amplitude strain, (τ_a = 0kPa) symmetric loading σ'_{vc} = 200 kPa

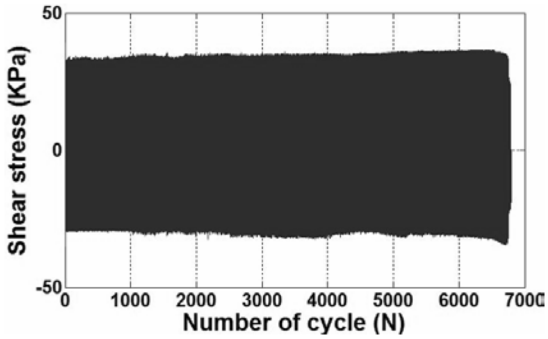

Fig. 5b. Shear Stress versus number of loading cycles with CSR = 0.2 at 15 % Double Amplitude Strain, (τ_a = 0kPa) Symmetric loading σ'_{vc} = 200 kPa.

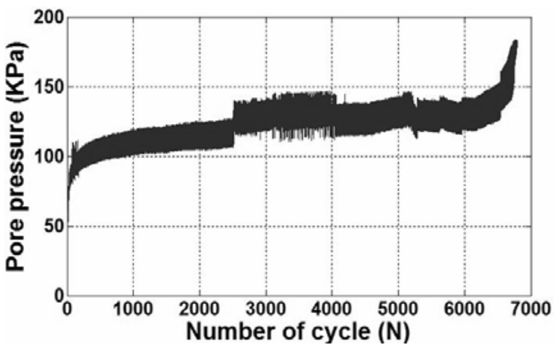

Fig. 5c. Excess Pore Pressure versus number of loading cycles with CSR = 0.2 at 15 % double amplitude strain, (τ_a = 0 kPa) symmetric loading σ'_{vc} = 200 kPa.

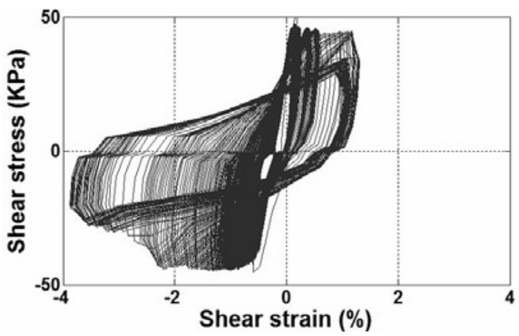

Fig. 6. Shear strain versus shear stress with CSR = 0.20 at 15 % double amplitude strain, (τ_a = 0 kPa) symmetric loading, σ'_{vc} = 200 kPa.

An important factor to consider in laboratory studies is whether the fabric produced by the method of sample preparation is similar to that found within the soil deposits being modeled. Numerous studies have shown that soil behavior is highly dependent upon laboratory sample preparation techniques for example (Milulis et al., 1977; Miura and Toki., 1982). According to Kuerbis and Vaid (1988) a reconstitutive sand sample preparation must fulfill the below criteria: 1) the method must be able to produce loose to dense samples in the density range expected within an in-situ soil deposit; 2) the samples must have a uniform void ratio throughout ; 3) the sample must be fully saturated, particularly for undrained testing; 4) the samples should be well mixed without particle size segregation, regardless of particle size gradation or fines content; and 5) sample preparation method should simulate the mode of soil deposition commonly found in the soil deposit being modeled. Several methods of sample preparation are available in the literature. Most commonly used sample preparations are air pluviation, water pluviation, slurry deposition, dry deposition, and moist tamping. The major factors which affect the relative density of air pluviated sands are height of particle drop (Vaid and Negussey, 1988) and rate of deposition (Miura and Toki 1982). Air Pluviation of well- graded sand is not as successful as air pluviation of well-sorted sand. The oldest laboratory reconstitution technique is moist or dry tamping of soil in layers (Lambe, 1951). The technique consists of placing the consecutive layers of specified thickness into a sample former tube, and tamping each layer flat with a specified force and frequency of tamping before the next layer is placed. The moist tamping produces loose to dense partially saturated samples which may be somewhat non-uniform with respect to density or particle size gradation. Several studies have been conducted to assess moist tamped sample uniformity, often with conflicting conclusions as to success of the method (Castro, 1969; 1982). Water pluviation sample preparation technique has been used by several researchers (Lee and Seed, 1967; Vaid and Negussey, 1984).

The water pluviation technique produces uniform sample of poorly graded sand (Vaid and Negussey, 1984; Chaney and Miulis, 1987). But, particle size segregation is an issue in water pluviation of well-graded or silty sand. Kuerbis and Vaid (1988) presented a sand sample preparation named as slurry deposition method. During this study, dry tamping approach or dry deposition approach is used. A sample weight of 100 g is used and laid in 3~5 layers in wire-reinforced membrane (diameter = 63.5 mm) to obtain the required relative density. Marine silty sand has minimum voids ratio of 0.74 and maximum voids ratio of 1.18.

3. Test results and discussions

A total of 18 CDSS tests were performed and influence of average and cyclic shear stress on cyclic shear strength is presented in the form of design graph or design diagram in Fig. 10.

Figure 5a shows development of shear strain with increasing number of loading cycles. The 15% of cyclic double apmlitude and/or permanent shear strain are taken to be a failure criterion, which has been widely used as a failure criterion (e.g., Anderson, 2009). In this particular loading case, the shear strain failure (γ_p = 0 & γ_{cy} = 15 %) is reached at around 6791 number of loading cycles.

Figures 5b and 5c show the behavior of the silty sand at symmetric loading. The primary mode of failure in the specimen is through liquefaction of the soil specimen as evidenced by the Pore pressure graph in Fig. 5c. The soil failed when the Pore pressure is substantially close to the initial vertical consolidation stress of 200 kPa.

The range of cyclic shear stress and average shear stress can be found in Table. 1. The corresponding cyclic shear strain and permanent shear strain are shown in the design graphs.

The observed noise in the data in Fig. 5c can be attributed to some errors caused by the membrane as large cycles may temporarily soften the confining stress provided by the wire membrane. However, the general trend of the graph still agrees with the expected behavior.

Figure 5c also shows that the general behavior of pore pressure development towards failure is to reach a value near the initial vertical confining stress. The observed value is around 180 kPa.

The graph in Fig. 6 shows the cyclic degradation of the silty sand. As the soil loses stiffness, a lower magnitude of shear stress induces a higher magnitude of cyclic strain.

Figure 7 shows the shear stress versus normal stress curve with CSR = 0.20, shear strain = 15 %, zero average stress or symmetrical loading, and vertical effective stress σ'$_{vc}$ = 200 kPa. Cyclic shear strain develops and practically no permanent shear strain is observed in symmetrical loading condition. It can also be noted that from the initial consolidation stress of 200 kPa, the remaining vertical effective stress in the specimen is around 20 kPa.

Figures 8a, 8b and 8c show the behavior of the silty sand when subjected to a combination of an applied average shear stress of 40 kPa and cyclic shear stress of 60 kPa. The prevailing mode of strain failure is evident in Fig. 8a as the accumulation of permanent shear strain reaches 15 %. Figure 8c shows that the excess pore

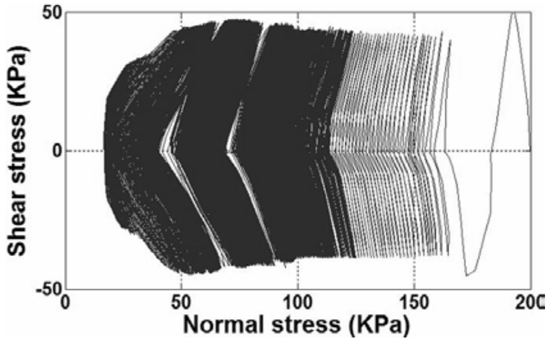

Fig. 7. Normal stress versus shear stress with CSR = 0.20 at 15 % double amplitude strain, (τ_a = 0 kPa) symmetric loading, σ'$_{vc}$ = 200 kPa.

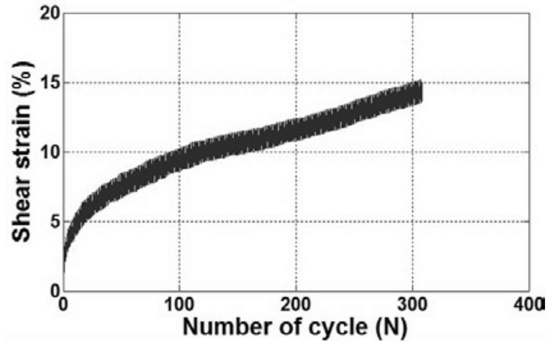

Fig. 8a. Shear Strain versus Number of Cycles with CSR = 0.30 at 15 % Double Amplitude Strain, Asymmetric loading(τa = 40 kPa), σ'vc = 200 kPa

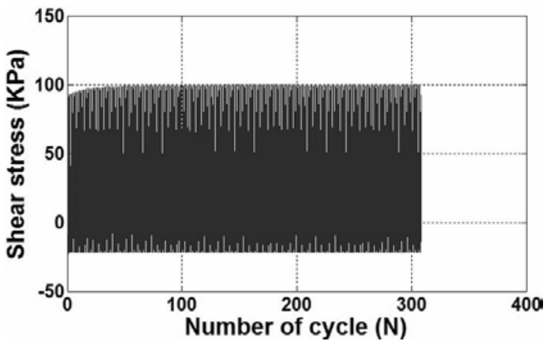

Fig. 8b. Shear stress versus number of loading of cycles with CSR = 0.30 at 15 % double amplitude strain, (τ_a = 40 kPa) asymmetric loading, σ'$_{vc}$ = 200 kPa.

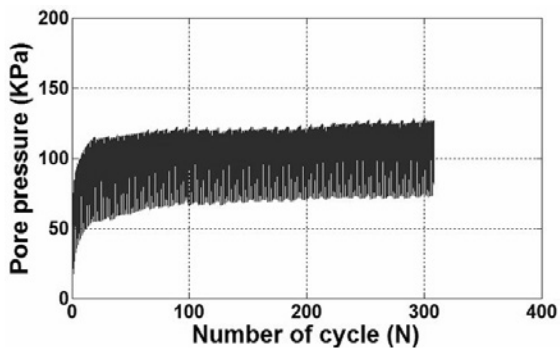

Fig. 8c. Pore pressure versus number of loading cycles with CSR = 0.30 at 15 % double amplitude strain, (τ_a = 40 kPa) asymmetric loading, σ'$_{vc}$ = 200 kPa.

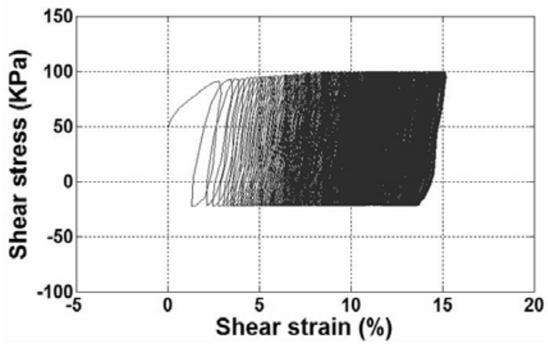

Fig. 9a. Shear stress versus shear strain with CSR = 0.30 at 15 % double amplitude strain, (τ_a = 40 kPa) asymmetric loading, σ'_{vc} = 200 kPa.

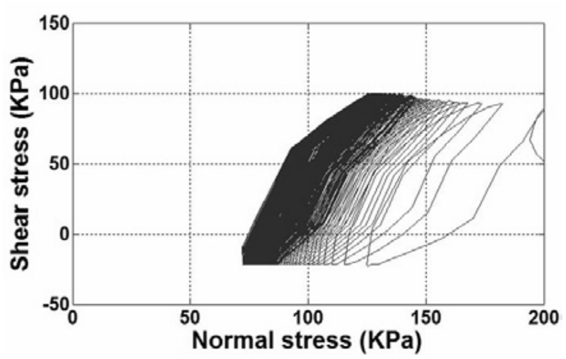

Fig. 9b. Shear stress versus normal stress with CSR = 0.30 at 15 % double amplitude strain, (τ_a = 40 kPa) asymmetric loading, σ'_{vc} = 200 kPa.

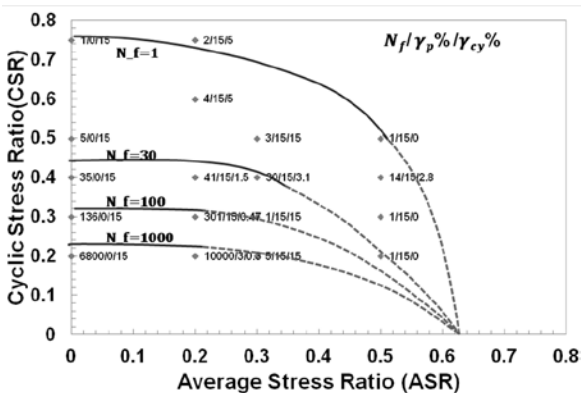

Fig. 10. contour lines for marine silty sand showing CSR and ASR and the corresponding cylic and permanent shear strains at failure.

pressure does not need to reach the initial confining stress of 200 kPa in order to fail.

Figures 9a and 9b further elaborate the failure mechanism of specimen subjected to asymmetric loading conditions. The strain behavior in Fig. 9 shows the progressive failure of the soil towards the 15% failure criterion.

Design diagrams from 18 CDSS tests are presented in Fig. 10. The location of various points is determined by the average and cyclic shear stresses under which the tests were run, and the numbers along each point represent the number of cycles to failure, N_f, and the permanent and cyclic shear strains at failure, γ_p/γ_{cy}. The design contour diagram shows that increase of cyclic shear stress generally decreases the number of cycles to reach the shear strain failure criteria. The contours are extrapolated and drawn to originate from the average shear stress ratio of 0.625 at the condition when cyclic stress is zero, based on the very few test data near that condition. The origin can be precisely determined through more static and low level cyclic shear stress tests in the future. As the average shear stress increases, the marine soil deposit fails by progressive or permanent shear strain failure mode. It is also found that when average shear stress ratio increases the cyclic shear strain component decreases and permanent shear strain component increases. The data obtained from these tests agree reasonably well with the data obtained by Andersen (1999; 2004; 2009).

$$ASR_{CSR=0} = \frac{\tau_a}{\sigma'_{vc}} = \tan\phi \qquad [1]$$

The convergence of the design contour the value of at ASR = 0.625 suggest that at the monotonic behavior, the silty sand has an angle of repose equal to 32 degrees.

4. Conclusions

Undrained cyclic behaviors of soils have been known to depend on the number of loading cycles, vertical effective stress, cyclic shear strain, relative density, and the combination of cyclic and average shear stresses. In order to evaluate the effect of average and cyclic shear stresses on the undrained cyclic behavior of marine silty sand sampled from West Coast of Korean peninsula, 18 CDSS tests were performed. The results are presented in the form of design graphs.

In case of symmetric loading condition, symmetrical cyclic shear strain develops and practically no permanent shear strain is observed. We must also note that the pore pressure development needed to trigger failure in symmetric loading cases require that the pore pressure reach a value substantially close to the initial vertical consolidation stress.

The results show that increase of cyclic shear stress generally decreases the number of cycles to reach the defined cyclic shear strain failure threshold of 15 %.

As for asymmetric loading condition, it can be seen that the accumulation of permanent shear strain is the primary mode of failure. The pore pressure development in the case of asymmetric loading shows that failure is reached despite the soil specimen still retaining a considerable amount of effective stress. The contribution of the cyclic shear component decreases as the Average Shear Stress ration increases.

From the results, it can be concluded that the cyclic shear strength significantly depends on the cyclic and average shear stresses, and a combination of both types shear stresses. Therefore, their effects on the cyclic shear strength should be evaluated and be taken into consideration in the design.

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Symbols and abbreviations

ASR	Average stress ratio
$ASR_{CSR=0}$	Average stress ratio when cyclic stress = 0
CDSS	Cyclic direct simple shear
CSR	Cyclic stress ratio
C_u	Coefficient of uniformity
D_{10}	Particle diameter at 10 % passing
D_{60}	Particle diameter at 60 % passing
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
G_s	Specific gravity
K_0	Horizontal stress coefficient at rest
K_0	KiloPascal
N	Number of loading cycles
N_f	Number of cycles to failure
USCS	Unified soil classification system
$\Delta\sigma'_v$	Change in vertical effective stress
Δu	Change in pore pressure
σ'_{vc}	Initial vertical consolidation stress
σ_r	Radial stress
τ_a	Average shear stress
τ_{cyc}	Cyclic shear stress
γ_{cyc}	Cyclic shear strain
γ_p	Cyclic permanent strain
φ	Internal friction angle of ground